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A Study of Factors Which Influence the Mechanical Properties of Deep Permafrost

By

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ABSTRACT

Drilling and well completion procedures in many areas within the Arctic Circle have been greatly influenced by the presence of over a thousand feet of permafrost consisting of layers of gravels, sands, silts, and clays containing permanently frozen ice.

In this study it has been shown that the shear strength of permafrost can be partitioned into a resistance to deformation of ice in the presence of natural soil constituents, and a resistance to deformation of soil constituents under confining stress.

Studies using a dilatometer have indicated that the following factors affect the fraction of water which is in the ice phase:

1. Temperature
2. Pressure
3. Electrolyte concentration
4. Mineralogy
5. State of clay hydration

These same factors have been shown to influence the uniaxial strength of frozen soil samples. For all frozen samples studied at a fixed temperature, the logarithm of the maximum uniaxial strength varies linearly with the logarithm of the strain rate. Experimental techniques were developed so that creep behavior could be estimated at strain rates as low as about 10^{-9} min.$^{-1}$.

The elastic and flow behavior of soil constituents can be determined with thawed samples provided the soil is brought to the proper initial state and provided proper boundary conditions are imposed during deformation. For granular soils of high permeability and containing a high percentage of sand, mechanical behavior was independent of strain rate over the range of low strain rates investigated. On the other hand, stress level was an important variable. When compacted, soil samples initially exhibited elastic behavior. Ultimate strengths were limited by shear failure which was described quantitatively by a Mohr envelope. For undrained deformation, flow behavior was significantly influenced by mineralogy.

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Samples containing clay and weak soil constituents generated high pore pressures during deformation, thus lowering net soil stresses and resulting in low overall shear strengths. The amount of pore volume drainage was an important variable.

Tests of samples frozen under stress confirmed that the total resistance to deformation could be calculated by adding the resistance to deformation of the ice phase in the presence of soil constituents and the elastic and flow resistance of the soil matrix.

INTRODUCTION

The discovery of commercial quantities of petroleum in arctic regions has led to substantial changes in drilling and completion practices. One of the challenges that must now be faced by drilling engineers is that of dealing with permafrost. At Prudhoe Bay, for example, completion procedures will be greatly influenced by the presence of nearly 2000 feet of permafrost consisting of layers of gravels, sands, silts, and clays which are permanently frozen. Studies of the thermal behavior of wells completed in permafrost,[1-3] have indicated that some thawing around wellbores is generally to be expected during drilling and production operations. Other studies[4, 15] have shown that pressures will rise as saturated, thawed regions around a wellbore are refrozen. The level to which external pressures rise will be significantly influenced by the elastic and flow behavior of the surrounding frozen soil. There are a number of publications[5-13] dealing with the mechanical behavior of frozen soils. Much of the civil engineering literature deals with behavior of shallow permafrost at relatively low stress levels. Chamberlain,[14] however, has reported behavior at very high stress levels which might be of interest for nuclear excavation or detection. This paper reports a study of the factors which influence the mechanical behavior of deep permafrost. The influence of these seven factors will be discussed:

1. Confining stress
2. Pore pressure
3. Mineralogy
4. Electrolyte concentration
5. Temperature
6. Strain
7. Strain rate

THE INTERACTION OF ICE AND SOIL

Since permafrost is a composite material, we should expect it to exhibit mechanical characteristics similar to its constituents, ice and soil. A formal approach to ice-soil interaction is enlightening. Fig. 1 illustrates a cylindrical element of permafrost subjected to a total axial stress, S_a, and a total lateral stress, S_ℓ. Some fraction of the axial stress, σ_{as}, is carried through the soil matrix, and the remainder, σ_{ai}, is carried through the ice phase.

$$S_a = \sigma_{as} + \sigma_{ai} \quad \ldots\ldots\ldots\ldots\ldots (1)$$

Similarly, the total lateral stress can be divided into that portion carried by the soil matrix, $\sigma_{\ell s}$, and a portion carried through the ice phase, $\sigma_{\ell i}$.

$$\sigma_\ell = \sigma_{\ell s} + \sigma_{\ell i} \quad \ldots\ldots\ldots\ldots\ldots (2)$$

The uniaxial strength (i.e., when the sample is loaded along the axis but with no lateral force applied) of pure multicrystalline ice samples is known[11] to be a function of temperature, strain rate, and strain. It has been reported in the literature,[16] and confirmed in our laboratories up to lateral stresses of 1000 psi, that for ice under triaxial stress, the difference between axial and lateral stress is essentially a function only of temperature, strain rate, and strain. The small effect on strength of suppression of the freezing point by pressure can be accounted for by correlating strength with temperature measured below the freezing point.[16]

$$\sigma_{ai} - \sigma_{\ell i} = \sigma_{ui} \quad \ldots\ldots\ldots\ldots\ldots (3)$$

σ_{ui} = uniaxial strength of multicrystalline ice, a function only of temperature below the freezing point, strain rate, and strain.

For permafrost, the uniaxial strength must result from the resistance to deformation of ice in the presence of the soil matrix. For a given lithology, the uniaxial strength of permafrost was shown[5, 11] to be a function of temperature, strain rate, and strain as would be expected for the ice component.

The effect of hydrostatic pressure on freezing point and on water-soil interaction has been shown to be small at pressures up to 1000 psi. Uniaxial deformation or deformation of a triaxially confined sample would require essentially the same deformation history for the ice component, and we would, therefore, expect the relationship shown by Eq. 4.

$$\sigma_{ai} - \sigma_{\ell i} = \sigma_{up} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

σ_{up} = uniaxial strength of a permafrost sample, a function only of temperature below the freezing point, strain rate, strain, and lithology.

Substitution of Eqs. 2 and 4 into Eq. 1 and rearrangement yields Eq. 5.

$$(S_a - S_\ell) = \sigma_{up} + (\sigma_{as} - \sigma_{\ell s}) \ldots\ldots (5)$$

The difference between total axial and total lateral stresses during deformation of permafrost can thus be viewed as consisting of two components. The first component, which is time dependent, results from the resistance of ice to deformation in the presence of the soil matrix and for the range of conditions of interest is essentially equal to the uniaxial strength of the permafrost sample. This contribution to strength will be a function of temperature below the freezing point, strain rate, strain, and electrolyte concentration and will be influenced by the interaction of water with clay materials in the soil matrix.

The second component, which is essentially independent of time, results from the resistance of soil to deformation under the constraint of the imposed boundary conditions. This contribution will be independent of temperature over the small range of temperatures of interest for permafrost and will be insensitive to strain rate if loads are applied relatively slowly. This strength component will be a function of mineralogy and lithology, degree of compaction, stress level and strain.

In the following sections of this paper, permafrost and soil strengths and tests of Eq. 5 are described.

EXPERIMENTAL STUDIES OF FACTORS THAT INFLUENCE DEEP PERMAFROST BEHAVIOR

Consider first those factors which influence the resistance of ice to deformation. In natural permafrost, there are a number of variables which determine the fraction of water which will be frozen at a given temperature.[17-30] Two experimental approaches utilizing calorimeters and dilatometers for investigating these relationships have been mentioned in the literature. We have chosen to use a dilatometer which will operate at pressures up to 1000 psi and which is illustrated in Fig. 2. In using this equipment, a sample of thawed permafrost is frozen under pressure in a heavy walled vessel and cooled to a low temperature. The bath temperature is then raised in steps while holding the pressure on the system constant with automatic devices. At each selected temperature, sufficient time is allowed for the system to come to equilibrium. The volume change accompanying a change of temperature is measured to within 0.0025 cc. The volume change of the vessel and fluid (determined from a previous calibration) is subtracted from the total change of volume to yield the volume change of the permafrost sample. The permafrost volume change is corrected slightly for thermal expansion of permafrost components to yield the change of volume due to phase change. After subsequent analysis of the total water content, volume changes resulting from the phase change were converted into pounds of unfrozen water per pound of soil solid as a function of temperature. These studies indicated that the following factors affect the fraction of water which is in the ice phase:

1. Temperature
2. Pressure
3. Electrolyte concentration
4. Mineralogy
5. State of clay hydration

Preliminary studies showed that soil samples which had been dried but then reconstituted with water showed a smaller amount of unfrozen water than equivalent samples which had never been dried. All subsequent studies were conducted with preserved and fully hydrated soil samples.

Pressure suppresses the freezing point of water and also has a detectable effect on water-soil interaction as has been suggested in the literature.[18] The effect of pressure is illustrated on Figs. 3 and 4.

Analysis of a core taken through the permafrost at Prudhoe Bay has shown salt contents of the formation water varying from a few tenths of a percent to as high as 2.5 percent near the bottom of the permafrost. As water is converted into ice, the electrolyte remains in the liquid phase, thus concentrating and lowering the freezing point of the brine. Fig. 3 compares the electrolyte concentration effect for an "as received" sample containing 0.97 percent NaCl with the behavior of a leached sample containing approximately 0.12 percent NaCl.

One of the more important factors influencing unfrozen water content is the amount of clay or fine material present. It is likely that water interacts with clay surfaces to partially orient the water molecules. There will also be a capillary pressure effect in fine grain soils.[30] As the temperature changes, bound water will not exhibit a volume change that would be expected for unbound water. The water associated with soil constituents has been calculated from the dilatometer data. Values for fully hydrated samples of near-surface Prudhoe Bay permafrost soil and a typical drilling mud are shown on Fig. 4. Silt samples containing between 10 percent and 15 percent clays exhibited more bound water than sand and silty sand samples containing less than 5 percent clays. These values for associated water are of the same order of magnitude as values previously reported in the literature and which were obtained at low pressure.

The combined effects of porosity, pressure, salinity, and lithology on in situ ice content have been estimated for Prudhoe Bay and are shown in Table 1.

High Strain Rate Uniaxial Compression Tests

Factors which influence the maximum uniaxial compressive strength of recompacted, saturated permafrost samples are:

1. Temperature
2. Strain rate
3. Salinity
4. Lithology
5. Mineralogy

Three types of natural permafrost soils from Prudhoe Bay have been selected for investigation. Tables 2 and 3 give particle size distribution, density, mineralogy, and natural salinity. A sample preparation procedure was utilized which has been described previously.[5] Maximum uniaxial strengths are shown on Figs. 5, 6, and 7 for a temperature range from 8°F to 28°F and for strain rates from 5×10^{-5} min.$^{-1}$ to 5×10^{-2} min.$^{-1}$. The influence of salinity has been investigated further by preparing samples with interstitial water salinities of 0, 1, and 2 1/2 percent NaCl. Maximum uniaxial strengths of natural sand and silty sand samples containing waters of these salinities are shown on Figs. 8 and 9 at different strain rates and temperatures.

Uniaxial Creep Tests

When subjected to small shear stresses, ice within permafrost can creep at very low rates. Over many years, the cumulative deflection which can occur at low strain rates may be of engineering significance. The experimental study of creep presents practical difficulties because of the long periods of time required to gather data and because of the investment in equipment needed for running simultaneous experiments.

The creep of recompacted natural soil samples has been studied using a technique previously described.[5] The sequence of stress application has been modified, however, so that data can be obtained at lower strain rates in a practical amount of time. For synthetic sand-ice samples, the empirical relationship[5] between temperature, uniaxial stress, strain, and strain rate is given by Eq. 6.

$$\xi = A \left[\frac{\sigma}{1 - \left(1 - \frac{\epsilon}{\epsilon_m}\right)^2} \right]^B \quad \ldots\ldots (6)$$

$\epsilon \leq \epsilon_m$

ξ = strain rate, min.$^{-1}$

σ = stress, psi
ϵ = strain
ϵ_m = strain at which a minimum strain rate is achieved at essentially constant axial stress
A and B = empirical constants which are a function of temperature

To test the validity of this equation for natural soil samples, recompacted specimens were prepared from Prudhoe Bay soil and were tested at various temperatures and at constant strain rates over the range 5×10^{-2} min.$^{-1}$ to 5×10^{-5} min.$^{-1}$. Values of the constants A and B were determined from these data. Additional permafrost samples were then placed in loading frames and were subjected to a uniaxial stress sufficient to strain the sample about 1 percent at strain rates of about 10^{-5} min.$^{-1}$. These data were fitted to Eq. 6 to evaluate ϵ_m. The axial stress was then reduced by one half. Additional deflections were measured over a period of several weeks so that the strain rate could be established with reasonable precision. Subsequently, the axial load was increased in a stepwise manner to obtain data at intermediate strain rates. Results for recompacted natural Prudhoe Bay sand samples are shown on Fig. 10. These samples behaved in a manner similar to that previously reported for sand-ice cylinders. The straight line relationship appears to be valid to a strain rate as low as 3×10^{-9} min^{-1}.

Studies of Soil Behavior

Let us turn our attention now to the contribution of the soil matrix to permafrost strength. Eq. 5 suggests that this contribution can be measured with thawed samples provided the soil is brought to the proper initial state and provided proper boundary conditions are imposed during deformation.

We have studied the deformation behavior of thawed samples in a triaxial cell arranged for independent control of internal pore pressure. Recompacted samples were frozen in cylindrical molds and then transferred to a precooled triaxial cell. As samples were thawed, the pore space was kept saturated by injection of fluid. Selected initial stress conditions were imposed, and subsequently constant strain rate deformations were conducted. Most deformation experiments on the thawed samples were run without the removal or addition of pore liquid, thus giving what is usually called undrained deformation. Internal pressures were recorded continuously with a pressure transducer of negligible displacement.

The influence of the following factors has been considered:

1. Strain rate
2. Type of soil
3. Level of compaction
4. Stress level
5. Relationship between lateral and axial stress
6. Amount of pore volume drainage.

Vesić has shown that under high stress, unconsolidated sands will cease to compact after a few hours. Clays, on the other hand, may continue to compact for longer periods of time.[32] All of the natural soils tested in the course of this study contained a high percentage of sand and were of high permeability. Preliminary deformations were run at several strain rates. Mechanical behavior was found to be essentially independent of strain rate over the range of low strain rates investigated. Subsequent studies were conducted at a convenient strain rate of .00166 min.$^{-1}$.

Four types of soil have been studied, three natural soils described in Tables 2 and 3 as well as a 40 to 200 mesh pure quartz sand similar to that previously studied by several investigators.

Behavior of Pure Quartz Sand

Saturated quartz sand samples were hydrostatically loaded while maintaining the pore pressure about 10 to 15 psi lower than the external stress. Subsequently, the cores were axially shortened. Data for a typical, undrained deformation are shown on Fig. 11. The difference between axial and lateral stresses was small during the first few percent strain, and the internal pressure was relatively constant. After a few percent strain, however, the sand became dilatent and the internal pore pressure diminished. As the effective normal stress increased, the ability to support shear stress increased. As high shear stresses developed and with continued strain, the pore pressure as well as the undrained shear strength of the sample

approached relatively constant values. Undrained, deformation behavior at various stress levels is shown on Fig. 12. For samples prepared in a similar manner, the shear strength contribution of ice has previously been reported.[5] The algebraic addition of strength contributions suggested by Eq. 5 has been tested by adding the uniaxial strength of frozen sand-ice cylinders to the strength values shown on Fig. 12. Fig. 13 shows that the calculated total resistance to deformation is in good agreement with experimental values which were reported previously.[5]

Behavior of Thawed Natural Soils Under Stress

The effect of the initial state of stress in natural soils has been investigated. As an example of the stress range of interest, consider the in situ stresses at Prudhoe Bay. Density logs and cores have shown that the average vertical stress is about 0.89 psi/ft. The pore fluid pressure gradient just below the permafrost is about 0.45 psi/ft, and we assume that this was the pore pressure gradient at the time of permafrost freezing. A total horizontal stress gradient of 0.65 psi/ft, which is typical of unconsolidated granular soils, has been estimated. The average total stress gradient is thus estimated to be $(0.89 + 2(0.65))/3 = 0.73$ psi/ft. The ratio of pore pressure to average total stress is estimated to be about $(0.45/0.73) \approx 0.6$.

Samples of natural soils were prepared using the following procedure:

1. Recompacted samples were saturated and frozen in cylindrical molds with no confining stress.

2. Frozen samples were loaded into the triaxial cell and thawed. Liquid was injected at a slight pressure to keep the pore space saturated during thawing.

3. External stresses were increased hydrostatically while maintaining internal pressures about 10 to 15 psi below the external stress.

4. While maintaining the external stress at a desired value, the internal pressure was cycled between 60 percent and and 99 percent of the external stress.

5. In some cases, the axial and radial stresses were subsequently varied to subject the soil to shear. During this treatment the average external stress was held constant and the pore pressure was held constant at 60 percent of the average external stress.

6. After the initialization treatment, the core was shortened without addition or removal of pore liquid.

Preliminary experiments have shown that soil behavior is quite sensitive to the level of compaction. Most of the compaction effect is achieved upon the application of the first cycle of compaction stress. A slight additional strengthening was detected upon application of several additional cycles of compaction; we have arbitrarily subjected samples to 20 cycles of compaction. No additional effect was noted when samples were held under compacting stress for 24 hours before commencing the deformation test. Samples which were subjected to shear stabilization were somewhat stiffer during an initial elastic deformation, but the ultimate shear strength was unaffected. Data for several initial stress levels and for three types of soil are shown on Fig. 14. Initial stress level is an important variable, but all these natural soil samples exhibited about the same behavior.

In a separate set of experiments the effect of varying the relationship between lateral and axial stresses was studied. After subjecting the soil samples to the desired initial conditions, three types of experiments were run:

1. The lateral stress was held constant while the core was shortened.

2. The lateral stress was decreased by an amount equal to the increase in axial stress during shortening.

3. The lateral stress was decreased 1/2 of the amount of increase of the axial stress during shortening. This experiment was thus conducted at constant octahedral normal stress.

For fixed initial conditions, during shear failure of these soil samples the relationship between strain and $(S_a - S_\ell)$ was the same for each of the three experimental procedures described.

In these undrained tests with natural soil, the behavior of internal pore pressure was different from that exhibited by pure quartz sand which was illustrated on Fig. 11. During deformation of natural soils, internal pressures rose above their initial values, probably because of slipping or crushing of soil particles. Fig. 15 shows typical shear strengths and internal pressures for a natural silty sand sample, a strong quartz sand sample, and a 95 percent sand-5 percent clay sample. The increase in pore pressure exhibited by the natural soil and clay-sand mixture led to low net stresses and therefore low shear strengths.

Since total radial, total axial, and pore pressure were independently measured, net soil stresses could be calculated and plotted in the form of Mohr envelopes as illustrated on Fig. 16. As the shear stress was initially increased, behavior was elastic. For the natural soil sample, when the net shear stress approached about 200 psi, particle failure or slippage occurred, bringing the net stress onto the envelope of shear failure. Similar experiments with strong quartz sand showed elastic behavior which merged into shear failure as the envelope of shear failure was reached at much higher stress levels. During subsequent strain, the Mohr envelope could be determined over a broad range by varying the lateral stress level. The few experiments using samples prepared from 95 percent sand and 5 percent clay exhibited behavior similar to that shown by the natural soil samples.

In a separate set of experiments with natural soils, the pore pressure was held constant during deformation by removing pore liquid. During shear failure, the apparent angle of internal friction increased with strain, reaching 22 degrees at 5 percent strain and increasing to 28 degrees at 10 percent strain. The difference between axial and lateral stresses was increased above those values observed for undrained deformations, thus confirming that the amount of pore drainage is an important variable.

Elastic Behavior

The initial behavior of well compacted soil samples appeared to be elastic. Experiments at constant lateral stress permit the direct calculation of Young's modulus.

$$E = \frac{\Delta\sigma_a (1 + n\nu)}{\varepsilon_a} \quad \ldots\ldots\ldots\ldots (7)$$

E = Young's modulus, psi
ν = Poisson's ratio
$\Delta\sigma_a$ = change in axial stress, psi
ε_a = axial strain
$n = 0$ if $\Delta\sigma_\ell = 0$
$n = 1$ if $\Delta\sigma_\ell = -\Delta\sigma_a/2$
$n = 2$ if $\Delta\sigma_\ell = -\Delta\sigma_a$

Moduli measured at constant lateral stress were less than 3×10^5 psi but were a function of initial mean net stress as has been reported by other investigators.[29] Measured values of Young's moduli are shown on Fig. 17.

The volumetric compressibility of ice is[33] about 7.65×10^{-7} psi^{-1} and that of soil constituents is[34] about 6×10^{-8} psi^{-1}. The elastic modulus, Poisson's ratio, and average volumetric compressibility are related by Eq. 8.

$$C = \frac{3(1 - 2\nu)}{E} \quad \ldots\ldots\ldots\ldots (8)$$

Substituting measured values of the modulus and calculated average compressibilities into Eq. 8 shows that Poisson's ratio for undrained deformations will be essentially 0.5. If fluid is expelled from the sample during deformation, then the lower values of Poisson's ratio often mentioned for rocks and soils should be observed. Deformations of thawed, undrained samples conducted with variable lateral stress have verified the consistency of Eq. 7 when using a value of Poisson's ratio of 0.5. If a free gas phase is present, lower values of Poisson's ratio would be expected.

Behavior of Frozen Samples Under Triaxial Conditions

The triaxial cell was operated in a different manner so that samples could be frozen under stress. After subjecting thawed

samples to the desired initial conditions, the samples, while under stress, were frozen from the bottom. During freezing the internal pressure was maintained at 60 percent of the external stress by removing excess liquid at the top through a heated tube. Experimental results for several conditions are shown on Fig. 18. Behavior calculated with Eq. 5 is shown in this figure to be in good agreement with the experimental data.

CONCLUSIONS

1. The strength of permafrost can be partitioned into a resistance to deformation of ice in the presence of natural soil constituents, which is time dependent, and a resistance to deformation of soil constituents under confining stress, which is essentially independent of time.

2. The fraction of water which is in the ice phase is influenced by the temperature, pressure, electrolyte concentration, mineralogy, and state of clay hydration.

3. The shear strength of the ice in permafrost is influenced by the same variables mentioned above as well as being a function of the strain rate.

4. For natural Prudhoe Bay soils, the linear relationship between the logarithm of the maximum uniaxial permafrost strength and the logarithm of the strain rate is valid to a strain rate as low as 3×10^{-9} min.$^{-1}$.

5. The contribution of the soil matrix to the permafrost shear strength can be measured with thawed samples provided the soil is brought to the proper initial state and provided the proper boundary conditions are imposed during deformation.

6. Soil strength is insensitive to strain rate but is significantly influenced by type of soil, compaction and stress level, and condition of pore volume drainage.

7. Initial mechanical behavior of compacted samples is elastic.

8. The presence of clays or weak constituents in a natural soil limits the net shear stress that can be sustained by the natural soil during deformation.

NOMENCLATURE

A = empirical coefficient, Eq. 6
B = empirical exponent, Eq. 6
C = volumetric compressibility, psi^{-1}
E = Young's modulus, psi
P_i = interstitial pore pressure, psi
S_a = total axial stress, psi
S_ℓ = total lateral stress, psi
ϵ = strain
ϵ_m = strain at which a minimum strain rate is achieved at essentially constant axial stress
ν = Poisson's ratio
ξ = strain rate, min^{-1}
σ = stress, psi
σ_{ai} = axial stress acting through the ice phase, psi
σ_{as} = net axial stress acting through the soil, psi
$\sigma_{\ell i}$ = lateral stress acting through the ice phase, psi
$\sigma_{\ell s}$ = net lateral stress acting through the soil, psi
σ_{ui} = uniaxial strength of multicrystalline ice, psi
σ_{up} = uniaxial strength of a permafrost sample, psi

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TABLE 1 - ESTIMATED IN-SITU ICE CONTENT AT PRUDHOE BAY

Depth feet	In Situ Temperature °F	Average Ice Content lb. ice/cu. ft. soil
200	15.1	20.01
400	17.5	16.89
600	18.8	11.00
800	20.6	15.34
1000	22.3	15.34
1200	24.1	20.02
1400	25.7	20.82
1600	27.2	19.10
1800	28.9	8.62
1900	29.6	0

TABLE 2 - PARTICLE SIZE DISTRIBUTION AND DENSITY OF NATURAL PRUDHOE BAY SOIL SAMPLES

Description of Samples	Percent Retained on U. S. Sieve Mesh Size						Density of Recompacted Soil Saturated with Water lb./cu. ft.
	20	40	80	120	200	Less than 200	
Sand	25.6	24	42.2	2.0	1.7	4.4	133.0
Silty Sand	3	5.4	39.8	15.8	10.8	25.2	129.5
Silt	0.2	2.	8	5.4	8.6	75.8	115.1

TABLE 3 - MINERALOGY AND SALINITY OF NATURAL PRUDHOE BAY SOIL SAMPLES

Description of Samples	Salinity Percent NaCl in Formation Water	Weight Percent of Constituents								
		Quartz	Calcite	Dolomite	Kaolinite	Illite	Montmo-rillonite	Mixed Layer	Chlorite	Total
Sand	0.97	99	0	0	0	0	0	0	0	99
Silty Sand	0.15	75	15	1	0	4.4	0.7	0.8	0	99
Silt	0.07	62	23	2	0.9	3.9	0.3	3.1	2.4	98

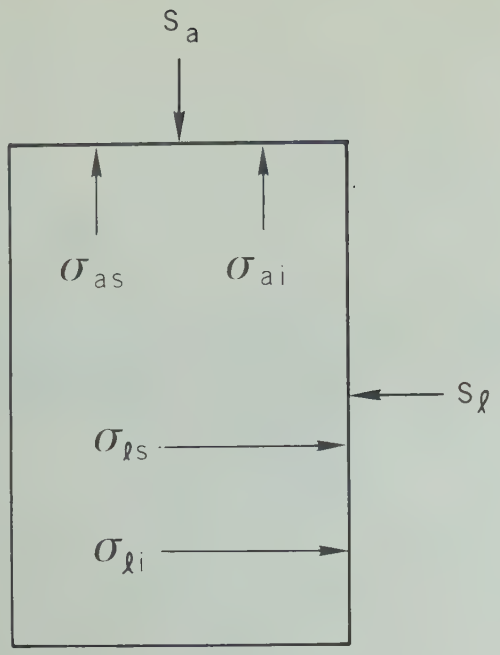


Fig. 1 - Stresses acting in a cylindrical permafrost sample.

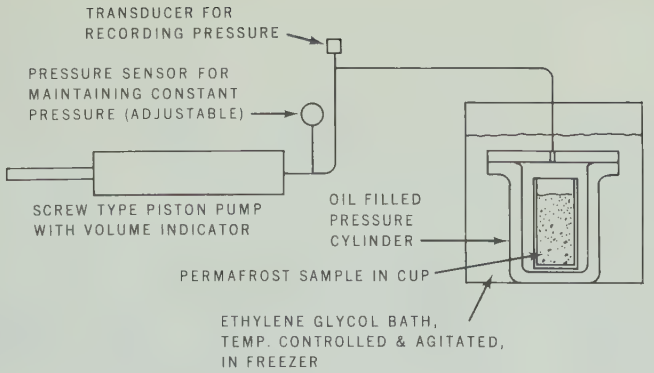

TRANSDUCER FOR RECORDING PRESSURE
PRESSURE SENSOR FOR MAINTAINING CONSTANT PRESSURE (ADJUSTABLE)
SCREW TYPE PISTON PUMP WITH VOLUME INDICATOR
OIL FILLED PRESSURE CYLINDER
PERMAFROST SAMPLE IN CUP
ETHYLENE GLYCOL BATH, TEMP. CONTROLLED & AGITATED, IN FREEZER

Fig. 2 - Apparatus for determining unfrozen water content.

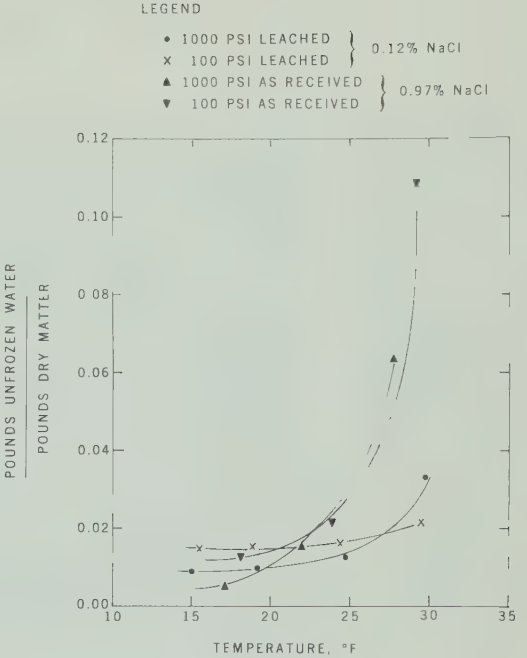


Fig. 3 - Unfrozen water content in a Prudhoe Bay near-surface sand sample.

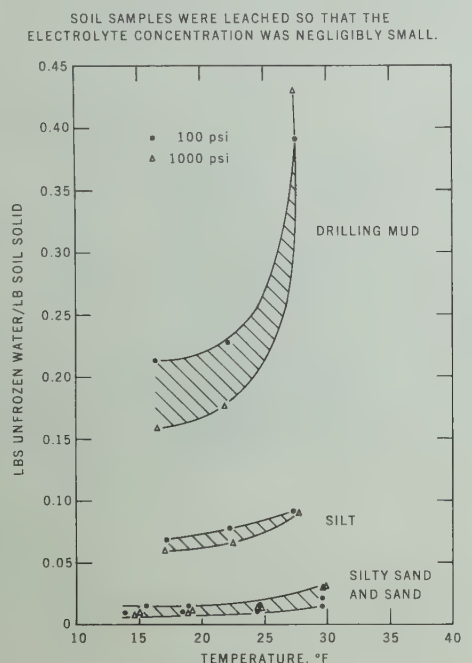

SOIL SAMPLES WERE LEACHED SO THAT THE ELECTROLYTE CONCENTRATION WAS NEGLIGIBLY SMALL.
• 100 psi
△ 1000 psi
DRILLING MUD
SILT
SILTY SAND AND SAND
LBS UNFROZEN WATER/LB SOIL SOLID
TEMPERATURE, °F

Fig. 4 - Unfrozen water content of drilling mud and natural Prudhoe Bay soils.

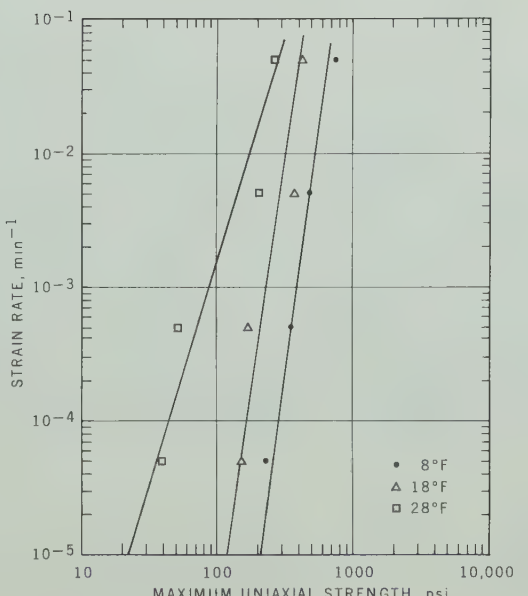


Fig. 5 - Maximum uniaxial strength of recompacted Prudhoe Bay sand. Before freezing the pore space was saturated with water containing 0.97 percent NaCl.

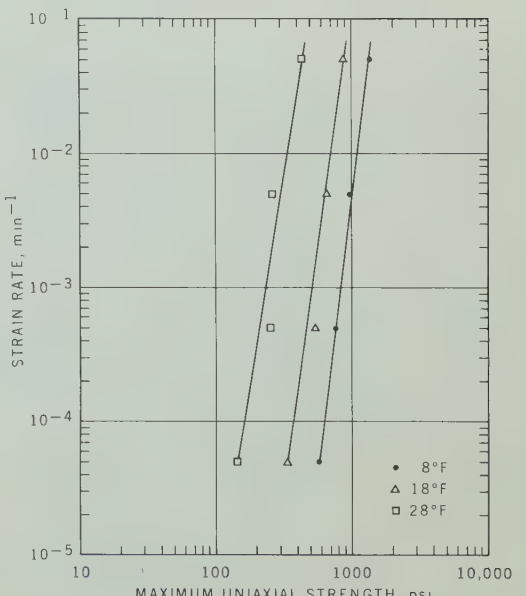


Fig. 6 - Maximum uniaxial strength of recompacted Prudhoe Bay silty sand. Before freezing the Pore space was saturated with water containing 0.15 percent NaCl.

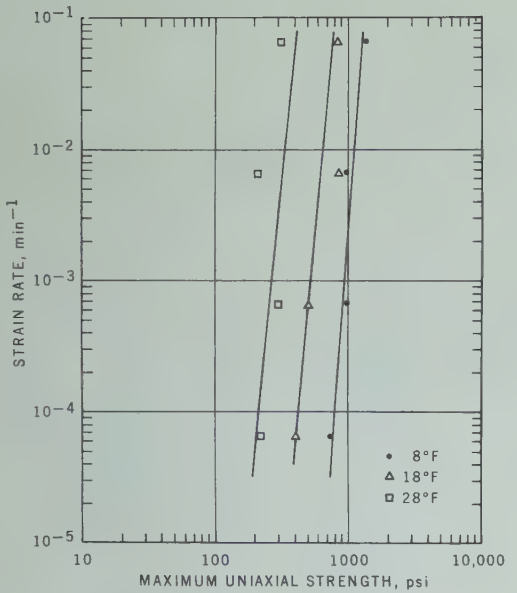


Fig. 7 - Maximum uniaxial strength of recompacted Prudhoe Bay silt. Before freezing the pore space was saturated with water containing 0.07 percent NaCl.

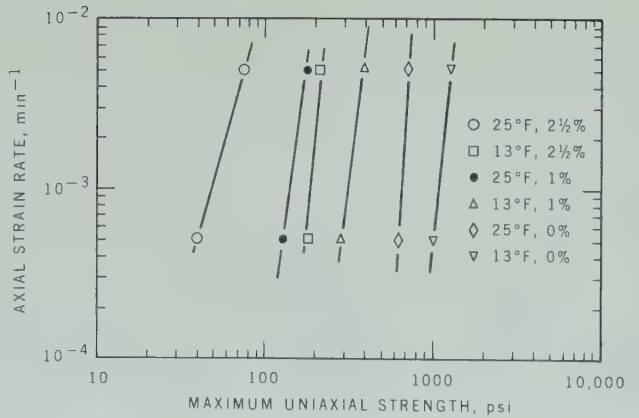


Fig. 8 - Influence of salt concentration on the uniaxial strength of a saturated, recompacted sample of natural Prudhoe Bay sand.

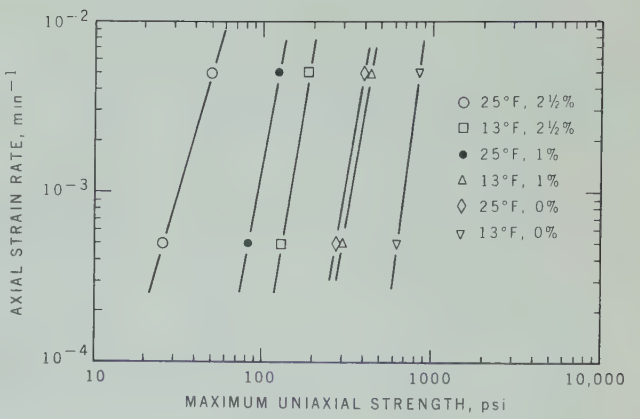


Fig. 9 - Influence of salt concentration on the uniaxial strength of a saturated, recompacted sample of natural Prudhoe Bay silty sand.

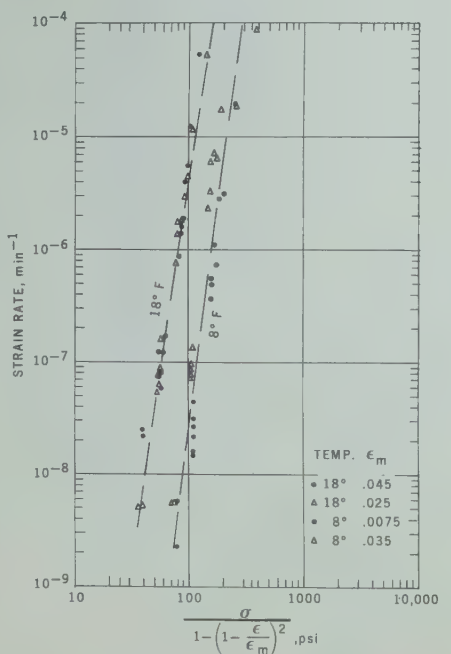


Fig. 10 - Creep behavior of recompacted Prudhoe Bay near-surface sand.

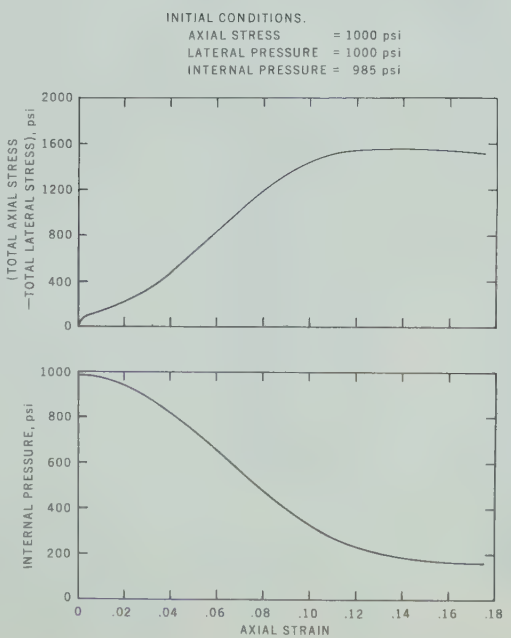

INITIAL CONDITIONS.
AXIAL STRESS = 1000 psi
LATERAL PRESSURE = 1000 psi
INTERNAL PRESSURE = 985 psi
(TOTAL AXIAL STRESS —TOTAL LATERAL STRESS), psi
INTERNAL PRESSURE, psi
AXIAL STRAIN

Fig. 11 - Deformation behavior of thawed undrained 40 to 200-mesh sand.

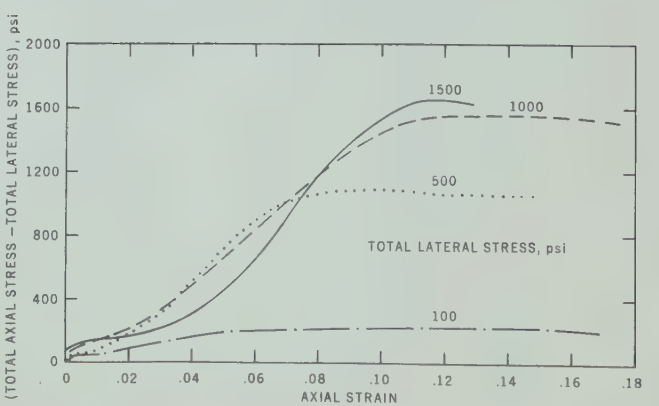
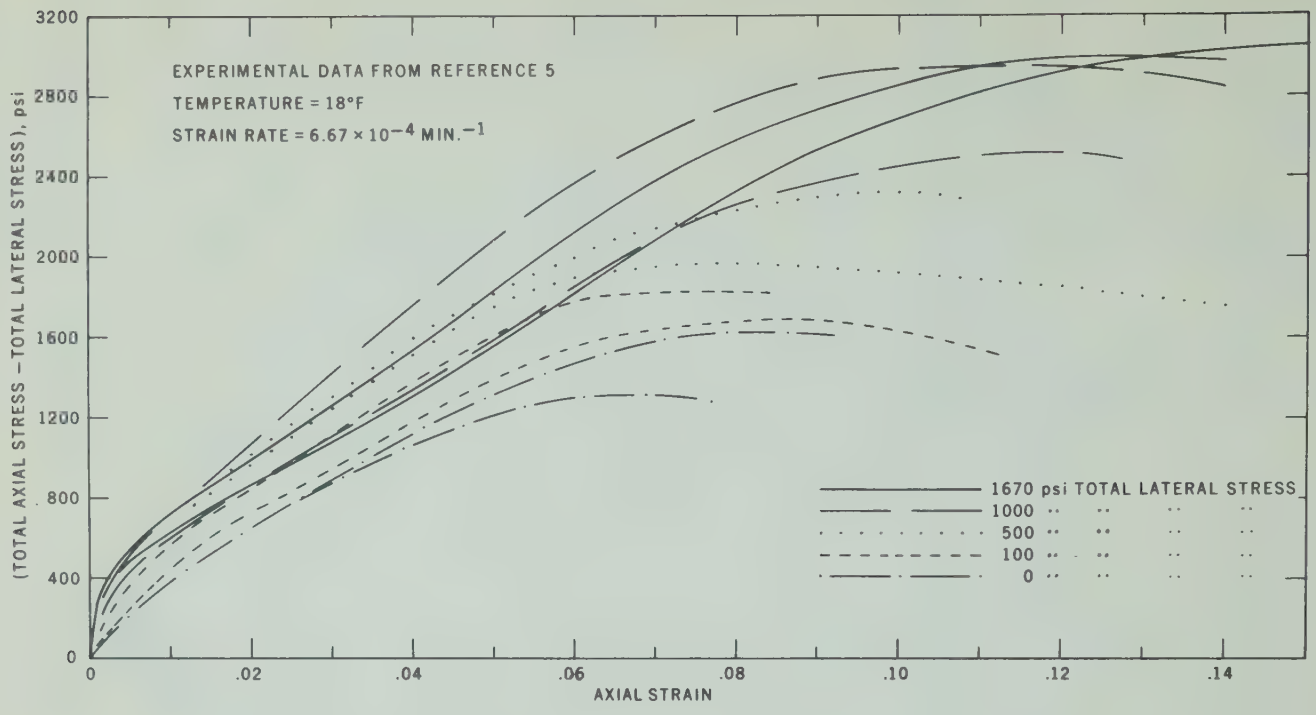


Fig. 12 - Behavior of thawed, undrained 40 to 200 mesh sand. Initial net compaction stress≈10 psi.



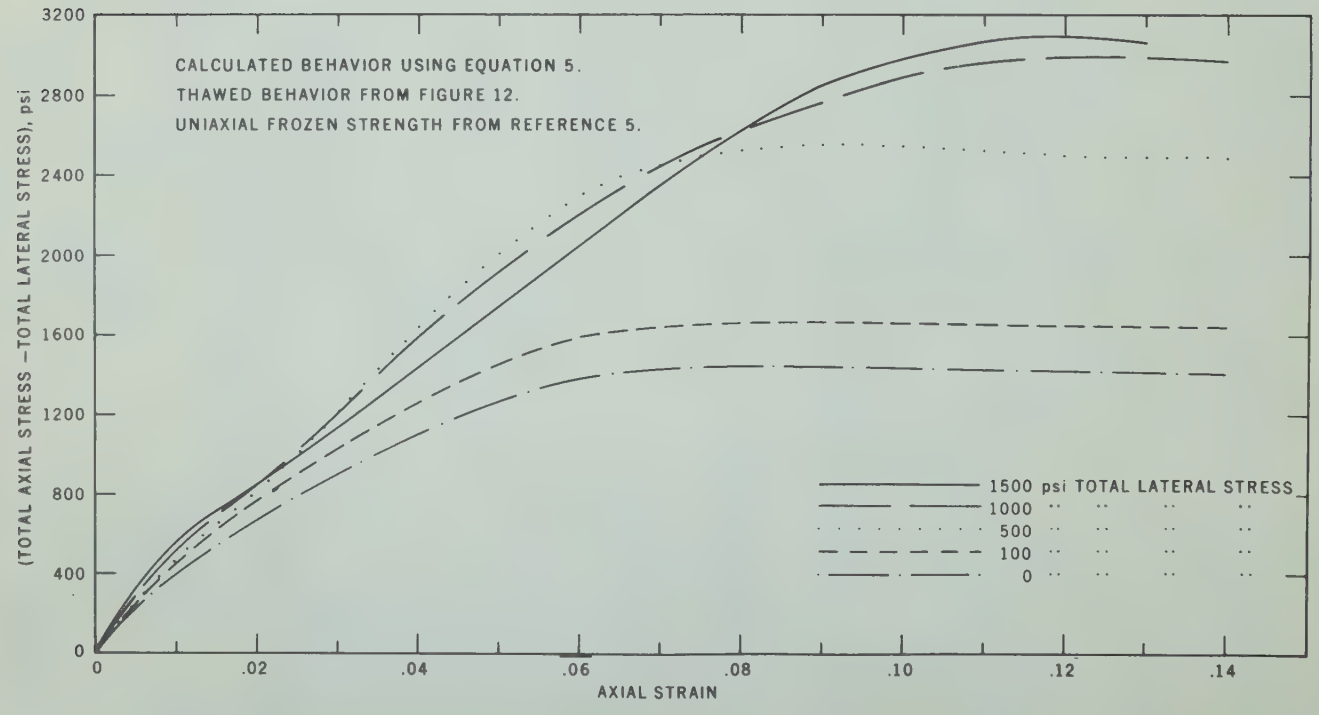


Fig. 13 - Comparison of experimental and calculated behavior of sand-ice samples under triaxial conditions. Samples frozen with zero confining stress.

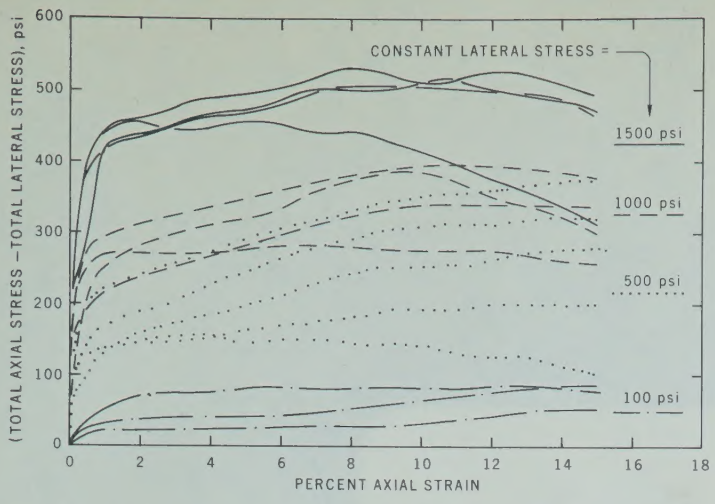


Fig. 14 - Shear strengths of natural Prudhoe Bay sands, silty sands and silts. During compaction internal pressure was cycled 20 times between 99 percent and 60 percent of external stress. Undrained deformations were conducted with the initial pore pressure equal to 60 percent of the lateral stress.

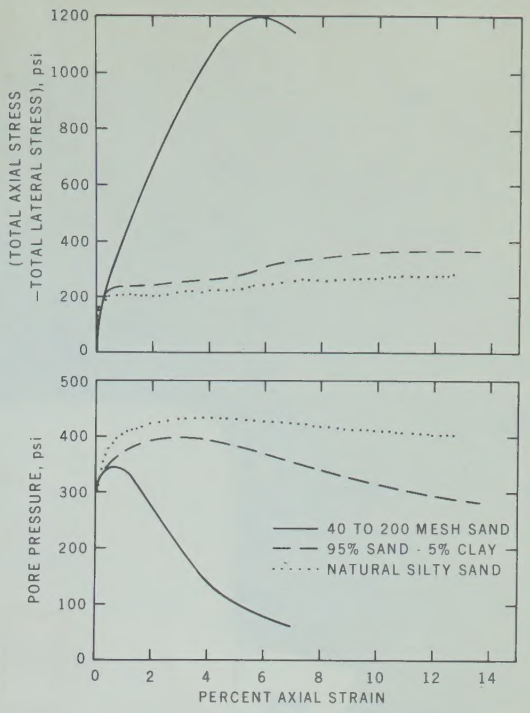

(TOTAL AXIAL STRESS – TOTAL LATERAL STRESS), psi
PORE PRESSURE, psi
40 TO 200 MESH SAND
95% SAND - 5% CLAY
NATURAL SILTY SAND
PERCENT AXIAL STRAIN

Fig. 15 - Effect of mineralogy on shear strength. All samples recompacted with 20 cycles. Total lateral stress held constant at 500 psi.

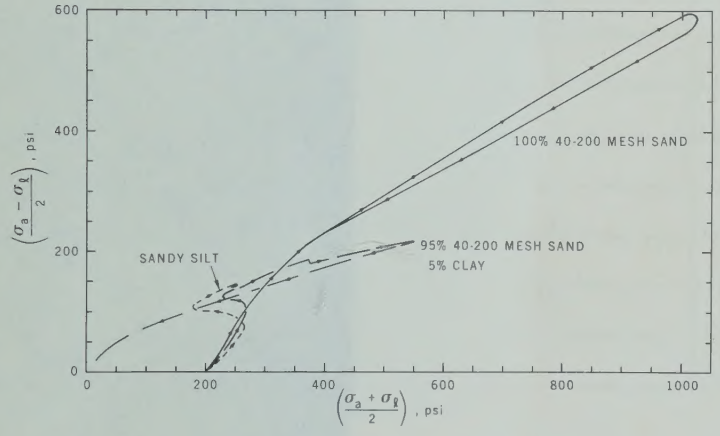


Fig. 16 - Mohr envelopes for various thawed soil.

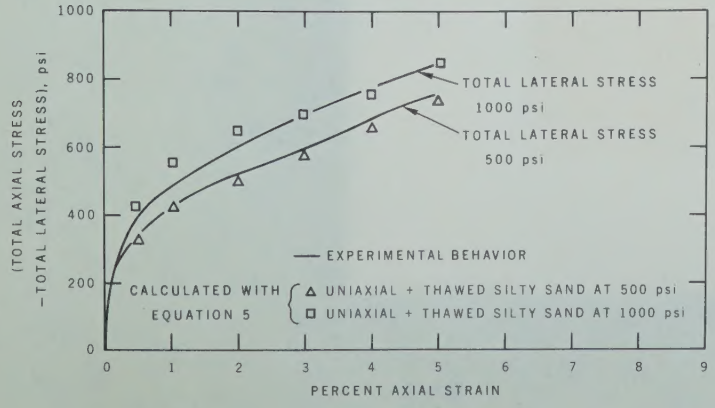

(TOTAL AXIAL STRESS – TOTAL LATERAL STRESS), psi
TOTAL LATERAL STRESS 1000 psi
TOTAL LATERAL STRESS 500 psi
EXPERIMENTAL BEHAVIOR
CALCULATED WITH EQUATION 5
UNIAXIAL + THAWED SILTY SAND AT 500 psi
UNIAXIAL + THAWED SILTY SAND AT 1000 psi
PERCENT AXIAL STRAIN

Fig. 18 - Comparison of yield behavior of frozen silty sand with summation of uniaxial and thawed behavior.

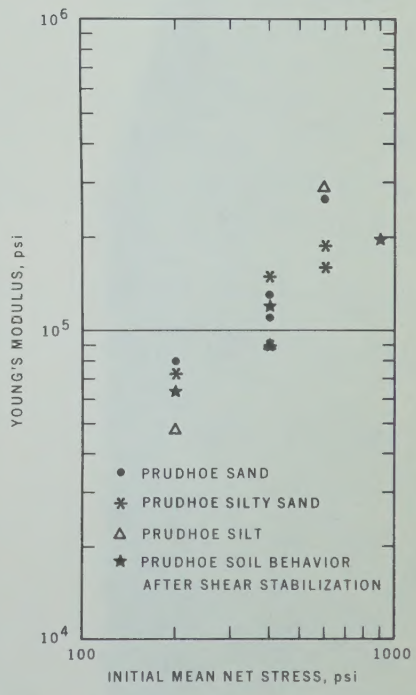


Fig. 17 - Young's modulus as a function of initial mean net stress. Samples compacted under the initial mean net stress with 20 cycles before beginning axial shortening.

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